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SUGGESTION TO DETERMINATION OF THE BEARING CAPACITY OF PILES ON THE BASIS OF CPT SOUNDING TESTS

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Abstract

The Cone Penetration Test (CPT) is well-recognized as a tool to calculate the ultimate bearing capacity of piles. Within the Hungarian physiographic territory, CPT and Static Pile Load Tests of the bored (CFA, protective tube) and driven (Franki) piles installed in different soils (gravel, sand and clay) were compared to determine the ultimate bearing capacity of piles using new formulae.

Keywords: bearing capacity of piles, CPT sounding test.

1. Backround

Both international and Hungarian professional literature ([1, 2, 6, 7, 8, 12, 11, 10]) deals intensively with the topic of the load bearing capacity of piles, determined on the basis of *in situ* exploration methods. This is a result of the rapid and extended proliferation of new exploration technologies which reveal more information about the underground condition on the spot (CPT and CPTu), than traditional boring methods did. Having in this way gained a great deal more knowledge about the soil through the new parameters, engineers try to develop appropriate formulas or equations that enable more efficient design and construction of structures. This also means that more reliable predictions can be made about the bearing capacity of a pile, at the beginning of the design stage.

The relevant professional literature arrived at the unanimous conclusion that nowadays the most informative method for the determination of bearing capacities of piles in granular soils is CPT (Cone Penetration Test) probing technology, because it differentiates between cone resistance (q_c) and local sleeve friction (f_s). The equipment produces the a diagram describing separately these two resistances, as a function of depth. An example is shown in *Fig. 1*.

In the Netherlands the design code [6] comprises the rules derived via innumerable cone tests and experiments for capacity calculations.

The load bearing capacity of the pile is determined from the cone resistance (q_c) of the CPT test. This is because the cone resistance values are more sensitive to variation in soil density than the sleeve friction, f_s and identification of the soil type from the ratio of q_c to f_s is not always clear-cut.

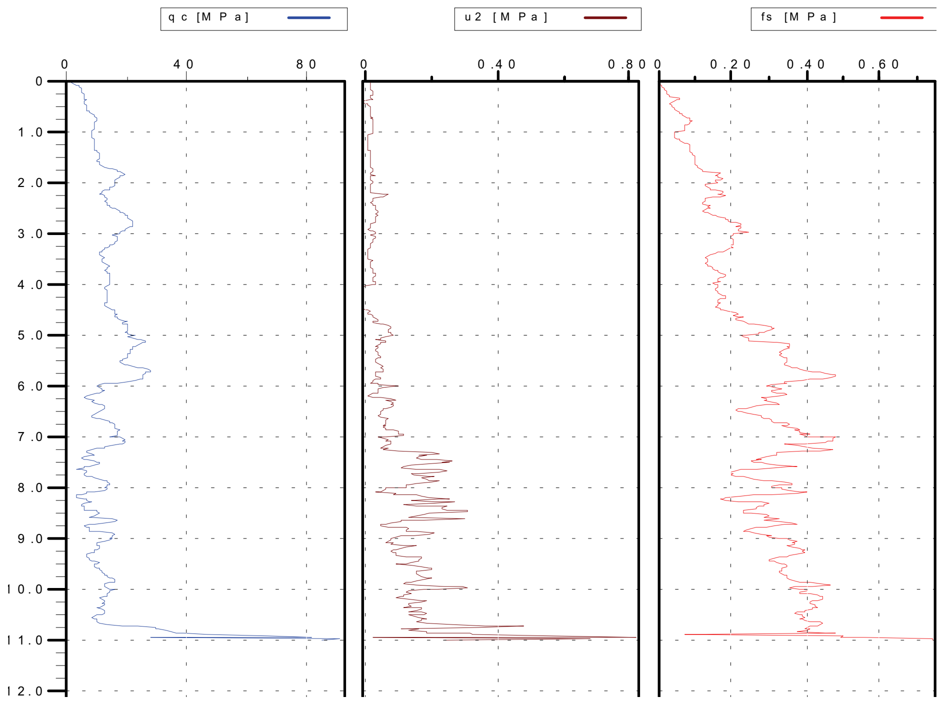

Fig. 1. CPT sounding diagrams

Consequently, in the traditional manner, the ultimate bearing capacity of a single pile (Q_u) is calculated as the sum of the ultimate resistance of the base (Q_b) and the ultimate resistance of the shaft (Q_s) capacities:

$$Q_u = Q_b + Q_s = A_b \cdot \overline{q_c} + U \cdot L \cdot \overline{\tau_s}$$

where:

A_b = nominal plan area of the base of the pile
U = length of the pile's periphery
L = length of the pile
$\overline{q_c}$ = average cone resistance in the zone of the pile toe
$\overline{\tau_s}$ = average ultimate skin friction along the pile shaft.

Based on experience MEIGH suggested to use the following correlation between pile skin friction and cone resistances:

Table 1. Values of skin friction

Pile type	Ultimate unit skin friction (τ_s)
Timber	0.012 q_c
Precast concrete	0.012 q_c
Steel displacement	0.012 q_c
Open ended steel tube + H-section	0.008 q_c
Open ended steel tube driven into fine to medium sand	0.0033 q_c

Values given in *Table 1* refer to piles that are exposed to static loads. MEIGH proposes to take the ultimate skin friction to 0,12 MN/m^2 at most.

The average resistance against the progress of the cone, or penetration ($\overline{q_c}$) can be derived using the formula:

$$\overline{q_c} = \frac{\overline{q_{c-1}} + \overline{q_{c-2}}}{2}$$

In the Netherlands, in accordance with the advice of MEIGH, [6] the generally applied method is where the average cone resistance ($\overline{q_{c-1}}$) is determined to the depth of four times the pile diameter (4D) below the toe, and the average cone resistance ($\overline{q_{c-2}}$) to the depth of eight times the pile diameter (8D) above the pile toe.

Regarding the 4D – 8D method, it is important to note that:

- the minor peak depressions have to be ignored from the calculation; supposedly they do not refer to thin weak strata, and
- also the $q_c > 30$ MN/m^2 values shall be ignored in this interval.

Obviously there are also methods other than the 4D – 8D method; in use they differ, however, only in the calculated depth below the pile toe (for example by taking 2D, instead of the 4D suggested above).

TE KAMP (1977)[9] preferred to suggest the safety factors presented in *Table 2*, for calculation of limiting capacity in the Netherlands, when the 4D – 8D method is used:

Because of the disturbance and loosening of the soil via the boring tool, the Codes advise not to use cone resistance values when the skin resistance of bored piles are calculated.

The relationship established for Dutch soil conditions is not necessarily applicable to cohesion less soils everywhere. The yielding and rupture of the soil caused by pushing a cone into the ground are different from those resulting from driving a pile by hammer followed by static loading. The work of VESIC (1977) [13] has shown the importance of the state of preconsolidation and mineralogy of the soil grains in any correlation of in-situ conditions with pile resistance. By

Table 2. Values of skin friction

Pile type	Factor of safety
Timber	1.7
Precast concrete, straight shaft	2.0
Precast concrete, enlarged shaft	2.5

coincidence static cone resistance in the Netherlands (and Belgium) was found to be equal to pile base resistance. Elsewhere GREGERSEN [4] found the pile base resistance to be only one half of the cone resistance for loose medium to coarse sands in Norway, and GRUTEMAN [5] reported that a factor of 0.75 is applied to the cone resistance to obtain the ultimate base resistance of piles in silty sands in Russia.

2. In-situ Tests

Analysis of in-situ test data can result in better design parameter estimates that will affect the ultimate bearing capacity, Q_u of piles. A comparison of Static Pile Load Test and CPT measurements in Hungarian soils was undertaken to better define mechanisms affecting Q_u and to create formulae that are appropriate for Hungarian soils and that also consider construction methods.

In this sense, the author selected the results of domestically performed Static Pile Load Tests where the results of the CPT tests were also available. Altogether data from seven CFA tests, three tests with protective tubes, and 26 Franki piles were gathered.

3. Suggested Formulae to Calculate the Ultimate Bearing Capacity of Piles

The derived formula with the values in *Fig. 2* relates to the failure load of a single pile. In deriving the formulae the customary static basis has been used as a starting point, whereby the ultimate bearing capacity of a single pile (Q_u) is the sum of the ultimate resistance of the base (Q_b) and the ultimate resistance of the shaft (Q_s) capacities:

$$Q_u = Q_s + Q_b$$

First part of the formula (Q_s) depends on the total surface area of the shaft; on the earth pressure acting thereon; on the interactive forces between the surrounding soil and the shaft; and on the technology of fabrication. These make:

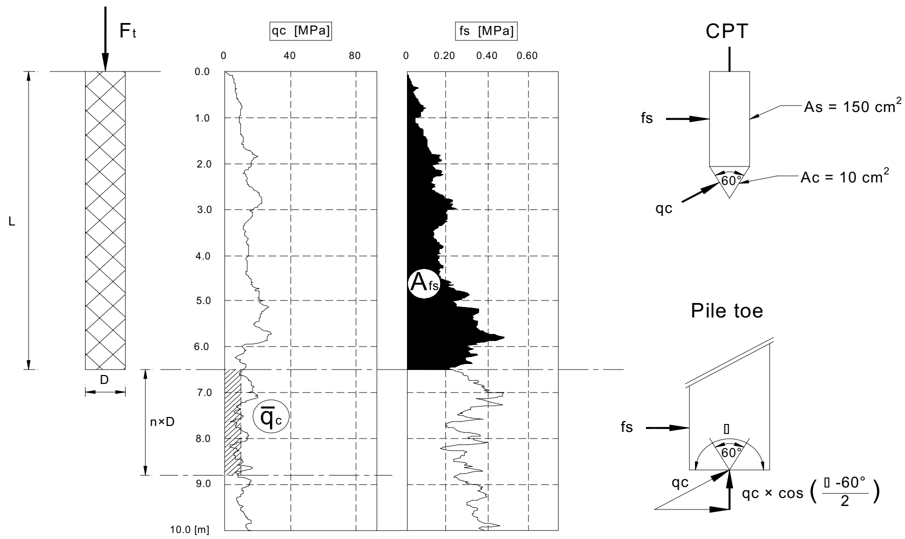

Fig. 2. Ultimate load bearing capacity of the pile using results of CPT testing

$$Q_s = \beta_s \cdot U \cdot L \cdot \overline{\tau_s} = \beta_s \cdot U \cdot A_{fs}$$

where:

- U = length of the pile's periphery
- L = length of the pile
- A_{fs} = area of the plotted f_s curve from the CPT probe test (*Fig. 2* explains)
- β_s = empiric factor with view on the applied piling technology; expresses the shaft resistance
- $\overline{\tau_s}$ = average ultimate skin friction along the pile shaft.

Second part of the formula (Q_b) depends on the extension of the surface area where the pile toe rests; on the specific resistance of the soil in the zone of the pile base; and on the applied piling technology. These make:

$$Q_b = \beta_b \cdot A_b \cdot \overline{q_c} \cdot \cos\left(\frac{\alpha - 60°}{2}\right)$$

where:

- A_b = nominal plan area of the base of the pile,
- β_b = empiric factor with view on the applied piling technology; expresses the base resistance,
- $\overline{q_c}$ = average value of the cone resistance below the pile toe (*Figure 1* explains). (It has been observed that the depth ($n \times$D) below the pile toe is strongly influenced by the applied piling technology, what has to be accounted for when the average q_c value is derived).

On the basis of piling technologies and pre-calculations the assumptions used and the conclusions are as follows:

- In the course of the calculations the bulb diameter for the Franki piles has been assumed to be equal with the trunk diameter; so the expansion of the bulb is included in the factor β_b.
- To account for the densification of the soil in the case of driven D = 60 cm diameter Franki piles in granular soils, it is recommended to use (higher than for the K_o = 1-sinφ equilibrium pressure) the values $\beta_s = 1.40$ and $\beta_b = 1.70$, as well as 3D zone-depth, in the calculations.
- For D = 60 cm diameter Franki piles in cohesive soils, it is recommended to use $\beta_s = 2.40$, $\beta_b = 2.70$, and 3D zone-depth.
- The values $\beta_s = 2.40$ and $\beta_b = 2.70$ are just one unit higher (because of the pore-water pressure) than in the case of granular soils.
- In the case of D = 100 cm diameter piles bored in protective tubes – presumably due to the accumulated pulverised sediment at the bottom of the hole – for the base resistance $\beta_b = 0.05$ and for the shaft resistance (lower than K_o = 1-sinφ) $\beta_s = 0.45$ and $1D$ depth-zone is recommended.
- For D = 80 cm diameter piles bored with CFA technology, it is recommended to use $\beta_b = \beta_s = 0.75$ and $2D$ depth-zone.

4. Analysis of the Ultimate Capacity of Piles Using the Static Pile Load Test

The pile load tests were performed according to the standard loading procedure described in the Hungarian Standards, MSZ 15005-1:1989 and MI 04.190:1984.

All pile load tests have been carried out until the failure load was reached.

5. Results of a Comparison of the Measured (Static Pile Load Test) and Calculated (CPT) Ultimate Bearing Capacities of Piles

The results of the recommended CPT method used to estimate the ultimate bearing capacity of the selected piles discussed in the In-situ tests section were compared with the results of Static Pile Load Tests results. Findings can be seen in the *Table 3*.

Based on the results of performed calculations, the regression coefficient (r) for each piling technology is as follows:

For D = 60 cm diameter Franki piles: $r = 0.87$,
For D = 100 cm diameter piles bored in protective tubes: $r = 0.84$,
For D = 80 cm diameter piles bored with CFA technology: $r = 0.94$.

Table 3. Comparison of calculated and measured bearing Capacity of piles

Location (ap=motorway)	Pile type	Length [m]	Diameter [m]	Soil below the toe	From CPT using the formulae β_s [1]	β_b [1]	n [1]	$Q_{u,\text{calculated}}$ [kN]	Static Pile Load Tests $Q_{u,\text{measured}}$ [kN]	Difference [%]
M3 ap./B 2		7.00	0.60	Gravel				3 694	3 650	−1%
M3 ap./B 3		5.00	0.60	Gravel				3 176	3 750	18%
M3 ap./H 29		7.00	0.60	Gravel				2 275	2 350	3%
M30 ap./1		9.50	0.60	Gravel	1.40	1.70	3	4 974	4 550	−9%
M30 ap./4		7.00	0.60	Sand				5 011	4 375	−13%
M3 ap./H 30	Franki	4.00	0.60	Sand				3 243	3 720	15%
M3 ap./B 9		6.50	0.60	Clay				3 820	4 350	14%
M3 ap./B 6		7.00	0.60	Clay				2 190	2 050	−6%
M3 ap./B 7		9.00	0.60	Clay	2.40	2.70	3	3 637	2 980	−18%
M3 ap./B 11		9.00	0.60	Clay				2 809	3 240	15%
M3 ap./B 13	Protective	23.00	1.00	Clay				2 867	2 600	−9%
M3 ap./B 14		17.80	1.00	Clay	0.45	0.05	1	2 116	2 100	−1%
M3 ap./H 32	tube	20.60	1.00	Clay				2 890	3 250	12%
M3 ap./HB 44		15.50	0.80	Sand				1 740	1 780	2%
M3 ap./HB 46	CFA	14.50	0.80	Sand	0.75	0.75	2	3 225	2 760	−14%
M3 ap./HB 47		13.50	0.80	Sand				2 665	3 050	14%
M3 ap./H 35		14.60	0.80	Sand				3 125	3 050	−2%
M3 ap./HB 42		15.80	0.80	Fine sand				2 160	1 927	−11%
M30 ap./2		13.80	0.80	Clay				4 023	3 900	−3%

6. Summary

This study presented the evaluation of a new method in predicting the ultimate bearing capacity of different piles (Franki piles, piles bored in protective tubes and piles bored with CFA technology) driven into different soils in Hungary.

Thirty six pile load test reports – with CPT soundings adjacent to the test pile – were collected. Prediction of pile capacity was performed on each pile; however, the statistical analysis and evaluation of the suggested prediction method was based on the results of the nineteen (presented in this paper) that plunged (failed) during pile load tests.

An evaluation scheme was executed to evaluate the CPT method's ability to predict the measured ultimate bearing capacity. Different values (β_s, β_b and n$\times$D) were suggested for different piling technologies for the evaluation scheme.

Based on the results of this study, the suggested formulae using results of CPT testing are given to predict the ultimate load bearing capacity of the piles.

While one may not expect that any calculation – carried out based on the result of either the CPT or of any other probing test – will lead in all cases straight to determination of the exact ultimate bearing capacity of pile derived using static loading test results, the performed study proves that more accurate approaches can be found to replace traditional static formulas in the design stage.

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